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SYNOPSIS

An experimental investigation was undertaken at Lehigh University to determine the fatigue strength of shear connectors for steel and concrete composite beams. Factorial experiments were designed to provide information regarding the effect of stress range and minimum stress level on the cycle life.

Included are fatigue tests of 35 push-out specimens having the concrete slab connected to the steel beam section by 3/4-inch stud connectors, 9 fatigue tests of push-out specimens using 7/8-inch stud connectors and 12 fatigue tests of push-out specimens using 4-inch 5.4 lb. channel connectors. The test data are described by mathematical equations which express the fatigue life as a function of the stress range.

Based on the reported fatigue tests and previous static and fatigue studies, a design criteria is proposed for the shear connectors of composite beams.

INTRODUCTION

Composite construction consisting of a concrete slab attached to steel beams by mechanical shear connectors is widely used for bridge spans of various lengths. Recent static[1] and fatigue studies[2][3] of composite members have indicated that the currently used design procoedure[4] for composite bridge beams is conservative. The wide use of this type of construction would indicate that considerable savings could be achieved by a better utilization of the connecting material.

The present design procedure for shear connectors is primarily based on static considerations.[5] The useful capacity of connectors was derived from static tests of beams with shear connectors and from push-out specimens by limiting the magnitude of slip to a value which would preclude the yielding of connectors. Design values are obtained by dividing the useful capacity by a suitable factor of safety which ensures that the ultimate strength of the member can be developed prior to yielding of connectors. Resulting designs were compared with available fatigue test results which indicated that fatigue failure was not a critical factor in the design. Since fatigue failure of connectors was not possible when this procedure was used, the spacing of connectors was determined from static load considerations. This results in a variable spacing of connectors which is proportional to the ordinate of the shear diagram.

Recent static studies have provided an approach for designing shear connectors so that the flexural strength of the member can be developed without requiring a limitation on the magnitude of slip or preventing yielding of the shear connectors.[1] This investigation revealed that the number of connectors required to develop the ultimate strength of a member could be reduced considerably when compared with the requirements in the AASHO Specifications. Also, the study showed that connectors need not be spaced in accordance with the intensity of statical shear to develop the ultimate strength. Uniform spacing of connectors was satisfactory for most loading conditions and neither ultimate strength nor deflections were appreciably influenced by the uniform spacing of connectors.

If the shear connector requirements are reduced by decreasing the factor of safety, fatigue failure of connectors may become the governing factor. Fatigue tests of composite beams at Lehigh University[(2)] and the University of Texas[(3)] revealed that no direct relationship exists between the static strength and the fatigue strength of connectors. Therefore, it is not advisable to retain the present design procedure and simply reduce the shear connector requirements. The test programs also indicated that when the number of shear connectors was adequate to prevent fatigue failure of connectors, the loss of interaction between slab and beam was not sufficient to cause appreciable increases in stresses and deflections in the beam. The initial fatigue studies did not provide complete information on the fatigue strength of connectors nor the effect of other variables on the fatigue strength.

The purpose of this investigation was to determine the fatigue characteristics of mechanical connectors for composite steel and concrete construction. Previous fatigue tests of composite beams had indicated that considerable variation could be expected in beam specimens, because it was difficult to assess the fatigue damage.[(2)(3)(6)] The failure of one or two connectors could not always be detected and did not significantly affect the beam behavior as the shear was redistributed to other connectors. Also, in beam tests it was not feasible to determine the fatigue behavior of connectors subjected to stress reversal.

Pilot studies indicated that push-out specimens yielded results comparable to beam tests so this type specimen was selected for the study. A push-out specimen had added advantages in that the loads to which the connectors were subjected could be more easily evaluated because redistribution was not significant. Also, a relatively large number of specimens could be tested more economically using push-out specimens.

EXPERIMENTAL STUDY OF 3/4-INCH STUD CONNECTORS

The principal phase of the investigation involved push-out tests of 3/4-inch stud connectors. The fatigue characteristics were evaluated by tests of 35 push-out specimens. Twenty-seven of these specimens formed the main experiment. Two were pilot tests and six additional specimens were added in order to supplement the data of the main experiment. Each specimen consisted of a 20 x 26-3/4 x 6 inch reinforced concrete slab attached by four 3/4 x 4 inch stud connectors to an 8 W 40 beam section as illustrated in Fig. 1. The studs were attached to the ASTM A36 steel beam sections by a local fabricator. All studs were inspected for soundness following the procedure as outlined in a draft of "Recommendations for Materials and for Welding for Steel Channel, Spiral, and Stud Shear Connectors", proposed by SubCommittee I of the ASCE-ACI Committee on Composite Construction dated July 10, 1964. The push-out specimens were tested by applying load to the edge of the reinforced concrete slab as indicated in Fig. 2. For stress reversal, load was applied to two edges of the slab as shown in Fig. 3.

The specimens for the main experiment were cast in groups of ten. All slabs were cast in a horizontal position as in a normal bridge structure. The same concrete mix proportions were used for each casting. Two cylinders were tested at the beginning of each fatigue test. The specimens were 28 to 92 days old at the time of testing. The mean compressive strength of all cylinders was 4300 psi and the standard deviation of the concrete strength was 335 psi.

The tests were conducted with an Amsler hydraulic pulsator and jacks at the loading rates of 250 or 500 cycles per minute. The rate of application of load was dependent on the specimen response. The average shear stress on the studs caused by the applied load was computed on the basis of the nominal cross-sectional area of the studs. Stress range is defined as the maximum horizontal shear stress minus the minimum horizontal shear stress in ksi on the cross-section area on studs or kips per inch of channel connector.

The main experiment was designed to evaluate two controlled variables: the stress range and the minimum stress. An outline of the main experiment design is given in Table 1. Five levels of maximum stress and stress range and three levels of minimum stress were selected on the basis of the previous beam experiments in order to establish the fatigue characteristics of the connectors for conditions that exist in bridge structures. Each minimum stress level was combined with three levels of maximum stress and stress range in such a manner that two complete 2 by 2 factorial experiments were included to obtain data on the effect of minimum stress on the maximum stress and minimum stress on the stress range. These four 2 by 2 factorial experiments are outlined by the dotted lines in Table 1. Three specimens were tested for each combination to provide replication.

Stress levels were assigned to the 27 specimens of the main experiment at random and the specimens were assigned to three test blocks (a, b, c) as indicated in Table 1. Within each test block of the 2 and 10 ksi minimum stress levels a random order of testing was followed. All three test blocks of the reversal specimens (-6 ksi minimum stress) were randomized since a separate test set-up was necessary. The random order of testing was followed to prevent variations caused by the controlled variables from being confounded with systematic variations due to uncontrolled variables such as the age of the specimens, behavior of testing equipment, etc.

The results of the main experiment indicated that range of stress rather than maximum stress was the more important variable. A stress range of 10 ksi appeared to be a suitable value for design. In an effort to obtain more data to supplement the main experiment, 6 additional specimens were tested with a stress range of 10 ksi and minimum stress levels of 2 and 10 ksi. This supplemental experiment is also shown in Table 1. The specimens for the supplemental tests were cast in one group. One cylinder was tested at the beginning of each fatigue test. The age of these specimens at the start of testing varied from 55 to 86 days. The mean compressive strength of all cylinders for this series was 3320 psi, and the standard deviation of the concrete was 109 psi. Hence, the additional test specimens also provided test data to help ascertain the influence of concrete strength on the fatigue life. In addition two pilot tests are reported which were conducted to aid in the experiment design. The total number of push-out tests of 3/4-inch diameter studs with minimum stress and range of stress as the major variables was 35.

EXPERIMENTAL STUDY OF 7/8-INCH STUD CONNECTORS AND 4-INCH 5.4 LB. CHANNEL CONNECTORS

The fatigue characteristics of 7/8-inch stud connectors was evaluated by push-out tests identical to those for the 3/4-inch stud connectors. Nine push-out specimens were designed and fabricated similar to the specimen illustrated in Fig. 1. The specimens were tested in the same manner as the 3/4-inch stud connectors.

The nine test specimens for the factorial experiment were all cast at one time. One cylinder was tested at the beginning of each fatigue test and yielded a mean compressive strength of 4470 psi and the standard deviation was 77.2 psi. The age of the specimens at the start of testing varied from 53 to 63 days.

The experiment design was identical to that for the 3/4-inch stud connectors except that only one test was made for each combination of stress conditions. An outline of the experiment design is given in Table 2.

The fatigue characteristics of 4-inch 5.4 lb. channel connectors was evaluated by tests of 12 push-out specimens. Nine of these specimens were part of the factorial experiment and three were pilot tests. Each specimen consisted of a reinforced concrete slab identical to that used for the stud connector specimens attached to the 8 W 40 steel beam section by two 6-inch lengths of 4-inch 5.4 lb. channels. One pilot test had the slab attached to the steel beam by only one 6-inch length of channel. Each channel was attached to the steel beam section by 3/16-inch fillet welds placed along the length of the heel and toe. The specimens were tested in the same manner as the 3/4-inch and 7/8-inch stud shear connectors.

The nine test specimens for the factorial experiment were all cast at one time. One cylinder was tested at the beginning of each fatigue test and yielded a mean compressive strength of 6045 psi with a standard deviation of 90.9 psi. The age of the specimens varied from 28 to 76 days.

The tests were all conducted with the Amsler hydraulic pulsator and jacks at the rate of 250 cycles per minute. The average force per inch of channel was computed by dividing the applied load by the total channel length. The load was applied to the test specimens of the main experiment by loading the edge of the concrete slab adjacent to the back face of the channel as this is the orientation that is commonly used on construction. The load was applied to the opposite edge of the slab during the pilot studies.

The experiment design was the same as for 7/8-inch diameter stud connectors and the outline of the main experiment is given in Table 3.

TEST RESULTS

All specimens were tested until failure occurred. For the stud shear connectors two different types of failure were apparent. Most of the fatigue failures were initiated at the reinforcement of the stud weld and penetrated into the beam flange causing a concave depression into the beam flange. In a few cases, the fatigue failure initiated at the reinforcement and penetrated through the weld. This latter condition was generally observed to occur when the weld penetration was incomplete. These typical failures are illustrated in the photographs shown in Fig. 4. The concave depression into the beam flange is apparent in Fig. 4b. Figure 4a shows failure through the weld. The crystalline texture of a typical fatigue fracture is readily apparent. The mode of failure was not a significant variable in these tests.

It was also apparent in the stud connector tests that two overall failure modes were evident for the push-out specimens. For the higher stress ranges and the lower minimum stress levels, the two studs nearest the applied load failed in fatigue. The remaining two studs were usually sheared off by the applied load as their ultimate static strength was exceeded before the machine could be stopped. For the lower stress range and higher stress levels, the applied load was more evenly distributed among the four studs and fatigue failures were evident in all four connectors.

For the channel shear connectors, the fatigue failure was generally initiated in one of the transverse fillet welds and propagated through the weld. In one instance, failure occurred in the channel web. No apparent stress raiser was noted for this case. With the channel connectors it was obvious that the channel nearest the applied load was carrying more load because the fatigue failure always initiated in this connector. The remaining channel was then pulled from the slab as the static strength was exceeded. Figure 5 shows typical fatigue fractures of the channel connector nearest the applied load and shows the remaining connector that was pushed from the slab as its static strength was exceeded before the machine could be stopped. The specimen on the left is from the main experiment while the specimen on the right is a pilot specimen.

Because of this observed behavior, an additional pilot specimen was fabricated which had only one channel connecting the concrete slab to the steel beam section. No significant difference was observed in the cycle life between the one or two channel connector push-out specimens.

The experimental results for the 3/4-inch stud connectors are summarized in Fig. 6 in which the stress range is given as a function of the logarithm of the number of cycles to failure for each minimum stress level. The test data from the main experiment are plotted as dots. The test data for the supplemental tests are plotted as circles and the pilot tests are plotted as crosses. The cycle life ranged from 27,900 cycles up to 10,275,900 cycles.

The experimental results for the 7/8-inch stud shear connectors are summarized in Fig. 7. The test data for S_{min} = -6 ksi are plotted as crosses, the data for S_{min} = +2 ksi are plotted as dots and the data for S_{min} = +10 ksi are plotted as circles. The cycle life ranged from 33,000 to 4,885,100 cycles for the 7/8-inch stud shear connectors. The experimental results for the 4-inch 5.4 lb. channel shear connectors are summarized in Fig. 8. The test data for S_{min} = -0.5 kips per inch are plotted as crosses, the data for S_{min} = +0.5 kips per inch are plotted as dots and the data for S_{min} = +1.5 kips per inch are plotted as circles. The cycle life ranged from 291,200 to 9,556,300 cycles for the channel shear connectors. The three pilot specimens of 4-inch 5.4 lb. channel shear connectors are plotted as triangles. The specimen having the single channel connector has a vertical line attached above the triangle. It is visually obvious that the fatigue strength of the single channel specimen was equivalent to specimens with two channel shear connectors. Also, the orientation of the channel connector whether facing toward or away from the applied load had no significant influence on the fatigue life.

ANALYSIS OF TEST RESULTS

The earlier studies reported in Refs. 2 and 3 had indicated that the fatigue strength of the stud connectors could be represented by a mathematical model of the form,

$$\log N = A + B\ S_r \qquad (1)$$

where S_r is the range of shear stress, N the number of cycles to failure and A and B empirical constants. The results of the beam tests are summarized in Fig. 9 where circles represent data for 1/2-inch diameter studs and dots represent data for 3/4-inch diameter studs. All specimens were tested with a low minimum stress level. No apparent leveling off of the S-N curves was noted in the beam tests. The mean regression curves in Fig. 9 were developed from data reported in Refs. 2 and 3. Also shown are the limits of dispersion of the test data. The failure criteria for the beam tests was taken as the initial fatigue fracture of the connectors. These tests indicated that 3/4-inch stud connectors had a lower fatigue strength than 1/2-inch stud connectors.

The factorial nature of the current experiments made possible independent determinations of the relative significance of the stress range and the minimum stress level. The analysis of the test data for 3/4-inch stud shear connectors showed that the slope of the S-N curves for each minimum stress level were not significantly different even at the 10% level. This indicated that the stress range affected the cycle life at each minimum stress level to the same degree. On the other hand, the analysis of variance indicated that the distances between the regression lines shown in Fig. 6 were significantly different even at the 1% level, i.e., the minimum stress was a significant parameter. Hence, stress range and minimum stress accounted for the variations in the experiment.

An examination of the test data for the main experiment and the test data for the supplemental tests indicates that the strength of concrete had only a minor affect on cycle life. The supplemental test specimens with a mean concrete strength of 3320 psi were near the lower limit of dispersion of the test data for specimens with a mean concrete strength of 4300 psi as can be seen in Fig. 6. This was in agreement with the earlier fatigue tests of beam and push-out specimens which had concrete strength varying from about 3000 to 6000 psi.[(2)(3)(7)(8)]

A further evaluation of the test data showed that the reason minimum stress had a significant effect on the cycle life was due to the stress reversal data. When all three curves are examined in Figs. 6 and 10 it is apparent that the stress reversal curve is some distance above the other two levels of minimum stress. In fact, an analysis of variance of the test data for minimum stress levels of 2 and 10 ksi indicated that there was no significant difference in the test data and that minimum stress was not significant even at the 10% level.

A third analysis was made of the test data which neglected the negative range of stress for the minimum stress level of -6 ksi. The data was analyzed considering the range of stress for this case to be from zero to maximum. This analysis indicated that when the negative range was neglected, minimum stress accounted for a small, barely significant portion of the fatigue strength and that stress range alone accounted for most of the variation in the experiment. The effect of minimum stress was significant at the 10% level but not at the 5% level.

Since the stress reversal specimens had significantly longer fatigue lives for the same stress range than the test data for 2 and 10 ksi minimum stress levels, it was concluded that a conservative estimate of the fatigue life could be obtained for all minimum stress levels by considering only the 2 ksi and 10 ksi minimum stress levels in the analysis. A regression analysis of the test data yielded Eq. 2.

$$\log N = 8.072 - 0.1753\, S_r \qquad (2)$$

where

S_r = range of shear stress in ksi, $S_{max.} - S_{min.}$

N = number of cycles to failure

The coefficient of correlation was 0.9327 and the standard error of estimate was 0.1940. The "goodness of the fit" may be judged from Fig. 10 where the test data are compared with Eq. 2 shown as the straight line. The equation appears applicable for cycle lives which vary from 10^4 to 10^7. Equation 2 was developed by neglecting the stress reversal data. The limits of dispersion were taken as twice the standard error of estimate and are shown as two dashed lines parallel to the regression line. It is readily apparent that such an analysis will provide a greater margin of safety for the stress reversal case. This is not considered to be critical as most connectors will be subjected to a shear loading in only one direction. Also, if shrinkage should occur, connectors designed for stress reversal may in fact be subjected to such a shear loading.

The results of the tests of 7/8-inch stud shear connectors were summarized in Fig. 7. An examination of the test data shows that the 7/8-inch stud connectors behaved similarly to the 3/4-inch stud connectors. Figure 11 compares the test data for the 3/4-inch and 7/8-inch stud connectors. The data for 3/4-inch connectors are shown as dots and the data for 7/8-inch connectors by circles. It is visually obvious and is verified by analysis that there are no significant differences between the fatigue strengths of 3/4-inch and 7/8-inch stud shear connectors.

The test data for the 4-inch 5.4 lb. channel shear connectors plotted in Fig. 8 also indicates that the stress reversal specimens had significantly greater fatigue strengths than the other two levels of minimum stress. Also, it is apparent that the test data for minimum stress levels of 0.5 k/in. and 1.5 k/in. are not significantly different. Hence, as with the stud shear connectors, a conservative estimate of the fatigue strength for all minimum stress levels can be obtained by neglecting the stress reversal data. A regression analysis of the test data for shear loading in only one direction yielded Eq. 3.

$$\log N = 8.9470 - 0.\overset{8}{\cancel{5}}444\ S_r \qquad (3)$$

The coefficient of correlation was 0.8648 and the standard error of estimate 0.1975.

Figure 12 compares the regression curve for the push-out specimens having stud connectors with the beam tests reported in Refs. 2 and 3. Figure 12 indicates that there is little difference between beam tests and push-out tests for 3/4-inch stud connectors up to about 300,000 cycles of loading. For longer cycle lives, the push-out tests yielded more conservative results. This behavior appears reasonable. In the beam test a loss of interaction was noted prior to connector failure.[2][3] Such a condition allows the connector forces to redistribute and results in a less severe stress condition than computed from elastic theory. In the push-out specimens the loading on the connectors is maintained at a reasonably constant level throughout the cycle life. Push-out tests therefore represent a lower bound for connector failure.

Because the push-out tests provide a lower bound it does not seem necessary to consider a value for design for stud shear connectors below the mean curve given by Eq. 2. A suitable design value can be obtained for any desired cycle life. For example, if the expected life is 2 million cycles, the resulting allowable stress range is 10 ksi. This value gives a suitable margin of safety with respect to beam test results.

On the basis of this data and rational, a tentative design formula for the allowable range of load can be obtained from Eq. 2. Equation 4 is the result.

$$q_r = 7850\ d_s^2 \qquad (4)$$

where q_r = allowable range of shear force per stud in pounds

d_s = diameter of the stud

Equation 4 has been developed from tests of 3/4-inch and 7/8-inch stud shear connectors. An examination of Fig. 12 indicates that it can be conservatively applied to smaller diameter stud shear connectors.

Equation 3 and the test data for the channel shear connectors are composed with small scale beam tests[(6)] in Fig. 13. The average shear stress range on the throat of the fillet weld is plotted as a function of the logarithm of cycle life. Only those test beams which had shear connectors similar in geometry to the channels in this experiment were considered. This seemed reasonable as an examination of standard channel sections showed that the thickness of the channel web was always equal or greater than the thickness at the toe of the flange. Since the size of welds would normally be governed by the flange toe thickness, beams reported in Ref. 6 with connectors which had the web area reduced so that the size of weld was greater than the web thickness were not considered applicable. In these latter tests the failure plane always occurred in the channel web.

It is readily apparent that channel shear connectors can be proportioned from the expected range of shear stress on the throat of the connecting fillet welds.

A tentative design formula for the allowable range of load for 2 million cycles was developed from Eq. 3 using the lower limit of dispersion. The lower limit of dispersion was used because of the limited amount of test data and the high concrete strength. Equation 5 is the result.

$$q_r = 2600\ w \tag{5}$$

where q_r = allowable range of shear force in kips per inch

w = length of a channel shear connector in inches measured in a transverse direction on the flange of a beam

DESIGN CRITERIA FOR SHEAR CONNECTORS

It is apparent from the results reported herein on the fatigue strength of shear connectors and from recent studies concerned with the ultimate load-carrying capacity of composite members[1] that a different design criteria is needed for the mechanical shear connectors used in composite bridge members. A rational philosophy of design should recognize that adequate static and fatigue strength is required in a bridge structure. Sufficient connectors should be provided to ensure the proper fatigue strength. In addition, it is necessary to provide sufficient connectors such that the static ultimate strength of the composite member can be achieved.

1. Fatigue Considerations

The magnitude of the shear force transmitted by individual connectors has been found to agree closely with values predicted by theory within the elastic range.[2] The connectors near the end of the beam are usually subjected to slightly higher stresses than connectors near midspan. However, the stresses on end connectors seldom exceed the values predicted by the elastic formula. For beams of normal proportions, the difference between the measured and predicted shear stresses is only a nominal amount.

Since fatigue is critical under repeated applications of working load, it is reasonable to determine the variation in shear stress using elastic theory. In other words the design criterion for fatigue is necessarily based on elastic considerations.

If complete interaction is assumed, the horizontal shear to be transferred by connectors for a given loading can be calculated from Eq. 6 as

$$H = \frac{Vm}{I} \qquad (6)$$

where H = horizontal shear per inch of length

V = shear in kips acting on the composite section

m = statical moment of the transformed compressive concrete area about the neutral axis of the composite section, $in.^2$

I = moment of inertia of the composite section, $in.^4$

In negative moment regions of continuous beams the value of m will be the statical moment of the area of reinforcing steel and the moment of inertia will be that of the steel beam and the reinforcing steel.

In simple span beams the range of shear stress throughout the span is dependent on the length of span. For spans up to about 70 ft. the range of shear varies from a maximum at the end of the span to about 85% of the maximum near midspan. For longer spans this variation is not nearly as great so that the range of shear is nearly constant throughout the span. This is illustrated by the shear envelopes plotted in Fig. 14. At the end of the beam the horizontal shear computed from Eq. 6 varies from zero to a maximum value as the live load moves onto the span. As is readily apparent from the shear envelopes plotted in Fig. 14, the range of horizontal shear stress will vary from zero to maximum at the supports to near full reversal at midspan depending on the span length. The dashed curves in Fig. 14 indicate the maximum shear envelopes for loads moving in the opposite direction.

For design, an average of the range of shears at the support and at midspan could be used to ascertain the required number of shear connectors where the range of shear is the difference in the minimum and maximum shear envelopes for passage of the vehicle.

An alternate, more conservative, yet simpler procedure would result by considering only the maximum shear at the support. In longer span bridges, the range of shear is more nearly uniform than in the shorter spans so that such an approach would be more conservative for the short span structures.

For continuous spans, the variation in the minimum-maximum shear envelopes along the lengths of the spans is usually somewhat greater than in simple spans. If the variation in the shear stress range is significant, a variable spacing of the connectors will be necessary. The range of stress on the connectors can be calculated using the properties of the cross-section which are applicable to the positive and negative moments and the appropriate shear range.

Since the fatigue investigation reported herein has indicated that stress range is the major variable influencing the fatigue strength of the shear connector, sufficient connectors can be provided for any desired cycle life.

The primary design consideration should be based on the fatigue criterion. Maximum and minimum shear stresses are computed from Eq. 6. The spacing of the shear connectors is given by

$$P = \frac{q_r}{H_{max.} - H_{min.}} \tag{7}$$

where $H_{max.}$ = maximum horizontal shear per inch as calculated from Eq. 6

$H_{min.}$ = minimum horizontal shear per inch as calculated from Eq. 6

q_r = allowable range of horizontal shear for the connector evaluated from Eqs. 4 or 5

P = spacing of shear connectors

(Note: The quantity $H_{max.} - H_{min.}$ is the range of shear stress. As is noted in Fig. 14, the range of shear at any location is V_r; the range of shear stress can be computed as $V_r m/I$. In continuous beams the range of shear stress is obtained by considering the sum of V_r^+ and V_r^- as indicated in Fig. 15.)

Equation 7 will determine the spacing in most designs. The spacing of connectors should never exceed 24-in. because connectors also perform the necessary function of holding the concrete slab in contact with the steel beam.

2. Flexural Strength Requirements

In addition to providing adequate fatigue strength, sufficient connectors should be provided to insure that the flexural strength of the composite member can be reached. Usually this requirement will be satisfied in most composite beams because fatigue considerations are usually critical except in cases of shored construction.

Recent research has shown that the flexural strength of composite beams can be achieved if sufficient connectors are provided to resist the maximum horizontal force in the slab.[1] This study also confirmed that connector spacing was not critical and that connectors could be spaced uniformly without deleterious effects.

At the ultimate moment of a composite beam, two stress distributions are possible as indicated in Fig. 15. Case I exists when the neutral axis is located in the steel beam. For either case, it is necessary to resist the horizontal force in the concrete slab. For the two cases, the maximum horizontal force is given by

$$H_1 = A_s F_y \tag{8}$$

$$H_2 = 0.85\, f_c' \, bt \tag{9}$$

Obviously only the smaller of these two forces must be developed in a simple span member. For continuous beams, an additional horizontal force due to the negative moment region is a tension force and is given by

$$H_3 = A_s^r \, F_y^r \tag{10}$$

where A_s = total area of the steel section including coverplates

A_s^r = total area of reinforcing steel in the slab at the interior support

F_y = minimum yield point of the type of steel being used

F_y^r = minimum yield point of reinforcing steel

f_c' = compressive strength of concrete at 28 days

b = effective width of the concrete slab

t = thickness of the concrete slab

Due to the redistribution of shear connector forces at ultimate load, the total ultimate strength of the connectors in the shear span is required to be equal to the sum of the horizontal forces acting at opposite ends of the shear span.[(1)] Figure 16 shows a free body diagram of a portion of the concrete slab between the point at which the ultimate flexural strength is developed and the support. It is apparent that the horizontal force in the slab must be resisted by the sum of the ultimate strengths of all connectors in the shear span.

In continuous beams the portion of the concrete slab between a point of maximum positive moment and a point of maximum negative moment should be considered as shown in Fig. 17. The sum of the ultimate strengths of the connectors in this region must equal or exceed the sum of the two horizontal forces acting on the slab.

Reference 1 has shown that the ultimate strength of shear connectors are given by the following expressions

Stud Connectors $$q_u = 930\ d_s^2\sqrt{f_c'} \qquad (11)$$

Channel Connectors $$q_u = 550\ (h + 0.5\ t)w\sqrt{f_c'} \qquad (12)$$

To ensure the development of the ultimate strength some margin should be provided to prevent premature failure of the shear connector. This can be achieved by providing a load reduction factor, $\emptyset$. It is suggested that a value of $\emptyset = 0.85$ should provide an adequate margin of strength. In order to check whether or not sufficient connectors are provided, one should first determine the horizontal force acting on the slab by selecting the smaller value obtained from Eqs. 8 and 9. For continuous beams the value obtained from Eq. 10 should be added to the smaller value given by Eqs. 8 and 9. The number of shear connectors required between the point of maximum moment and the support for simple beams or between the points of maximum negative and maximum positive moment for continuous beams is given by

$$N = \frac{H}{\emptyset q_u} \tag{13}$$

where H = horizontal force acting on the slab (smaller value from Eqs. 8 and 9 for simple beams and the sum of the smaller value and the value from Eq. 10 for continuous beams)

N = number of connectors between points of maximum and zero moments or the number between points of maximum positive and negative moments for continuous beams

q_u = ultimate strength of shear connector (given by Eqs. 11 and 12)

$\emptyset$ = load reduction factor

If the number of connectors given by Eq. 13 exceeds the number provided by the spacing given by Eq. 7, additional connectors should be added to ensure that the flexural strength is achieved.

SUMMARY AND CONCLUSIONS

1. Tests of 35 push-out specimens having the concrete slab connected to the steel beam section with 3/4-inch stud shear connectors, 9 tests with 7/8-inch stud connectors, and 12 tests with 4-inch 5.4 lbs. channel connectors were made to determine the fatigue behavior of the connectors.

2. A mathematical model expressing the logarithm of the fatigue life as a linear function of stress range was found to fit the test data. An analysis of variance indicated that minimum stress was a significant variable only for stress reversal. If the reversal portion was neglected the stress range was by far the most important independent variable.

3. The push-out specimen developed for this study provided test results directly comparable to beam tests for up to 300,000 cycles of loading. For longer cycle lives, the push-out tests yielded more conservative results because redistribution does not occur and the loading on the connectors is maintained at a reasonably constant level throughout the cycle life. Push-out tests therefore represent a lower bound for connector failure.

4. A design criteria for shear connectors is proposed which recognizes both the static and fatigue behavior of the shear connectors for composite steel and concrete members.

5. A design procedure is developed which provides a simpler and more economical design for the shear connectors of composite beams.

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TABLE 1. OUTLINE OF 3/4-INCH STUD CONNECTOR EXPERIMENT

MAIN EXPERIMENT

Maximum Stress (ksi) / Minimum Stress (ksi)	10	14	18	22	26
- 6	a 1 A b 1 A c 1 A	a 2 A b 2 A c 2 A	a 3 A b 3 A c 3 A		
2		a 2 B b 2 B c 2 B	a 3 B b 3 B c 3 B	a 4 B b 4 B c 4 B	
10			a 3 C b 3 C c 3 C	a 4 C b 4 C c 4 C	a 5 C b 5 C c 5 C

Stress Range (ksi) / Minimum Stress (ksi)	8	12	16	20	24
- 6			a 1 A b 1 A c 1 A	a 2 A b 2 A c 2 A	a 3 A b 3 A c 3 A
2		a 2 B b 2 B c 2 B	a 3 B b 3 B c 3 B	a 4 B b 4 B c 4 B	
10	a 3 C b 3 C c 3 C	a 4 C b 4 C c 4 C	a 5 C b 5 C c 5 C		

SUPPLEMENTAL EXPERIMENT

Maximum Stress (ksi) / Minimum Stress (ksi)	12	20
2	a 6 B b 6 B c 6 B	
10		a 6 C b 6 C c 6 C

Stress Range (ksi) / Minimum Stress (ksi)	10
2	a 6 B b 6 B c 6 B
10	a 6 C b 6 C c 6 C

TABLE 2. OUTLINE OF 7/8-INCH STUD CONNECTOR EXPERIMENT

Minimum Stress (ksi) \ Maximum Stress (kis)	10	14	18	22	26
- 6	e 1 G	e 2 G	e 3 G		
2		e 2 H	e 3 H	e 4 H	
10			e 3 I	e 4 I	e 5 I

Minimum Stress (ksi) \ Maximum Stress (ksi)	8	12	16	20	24
- 6			e 1 G	e 2 G	e 3 G
2		e 2 H	e 3 H	e 4 H	
10	e 3 I	e 4 I	e 5 I		

TABLE 3. OUTLINE OF 4 [5.4 LB. CHANNEL CONNECTOR EXPERIMENT

Minimum Stress (kips/in.) \ Maximum Stress (kips/in.)	3.0	3.5	4.0	4.5	5.0
-0.5	d 1 D	d 2 D	d 3 D		
+0.5		d 2 E	d 3 E	d 4 E	
+1.5			d 3 F	d 4 F	d 5 F

Minimum Stress (kips/in.) \ Stress Range (kips/in.)	2.5	3.0	3.5	4.0	4.5
-0.5			d 1 D	d 2 D	d 3 D
+0.5		d 2 E	d 3 E	d 4 E	
+1.5	d 3 F	d 4 F	d 5 F		

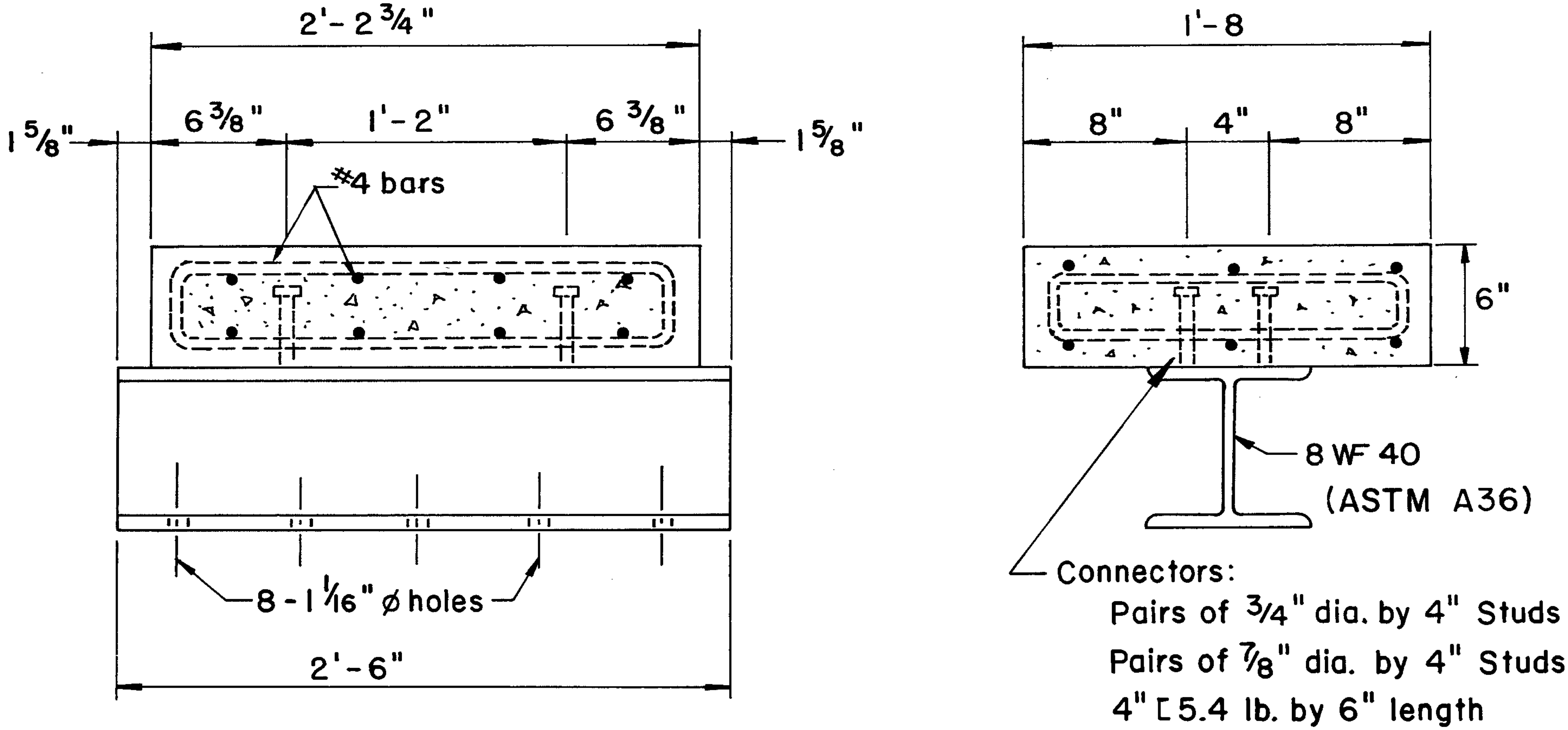


Fig. 1 DETAILS OF TEST SPECIMEN

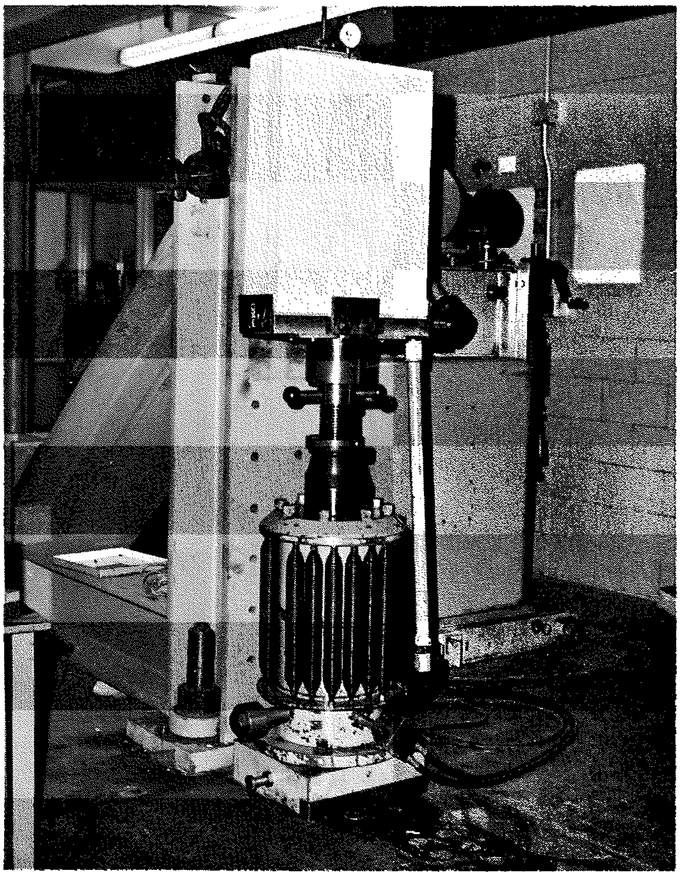


Fig. 2 TEST SETUP FOR LOADING IN ONE DIRECTION

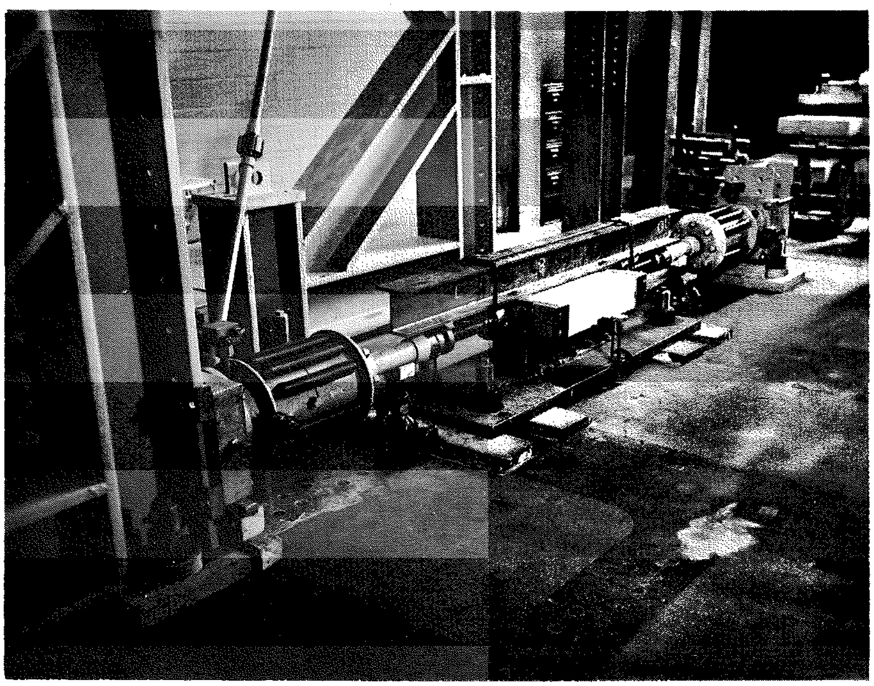
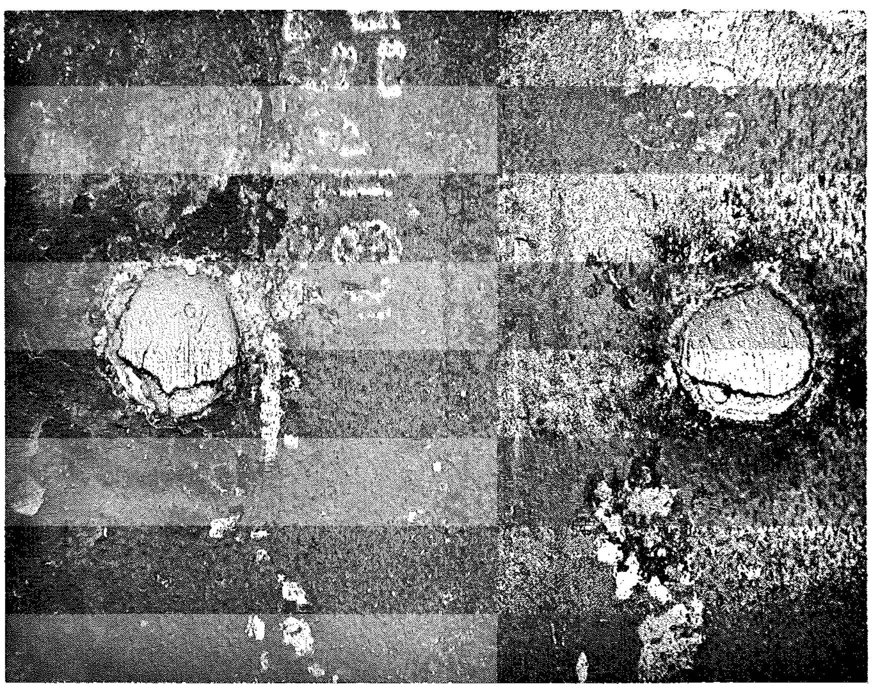


Fig. 3 TEST SETUP FOR STRESS REVERSAL TESTS

(a)



(b)

Fig. 4 TYPICAL FAILURES OF 3/4-INCH DIAMETER STUDS

Fig. 5 FAILURES OF CHANNEL CONNECTORS

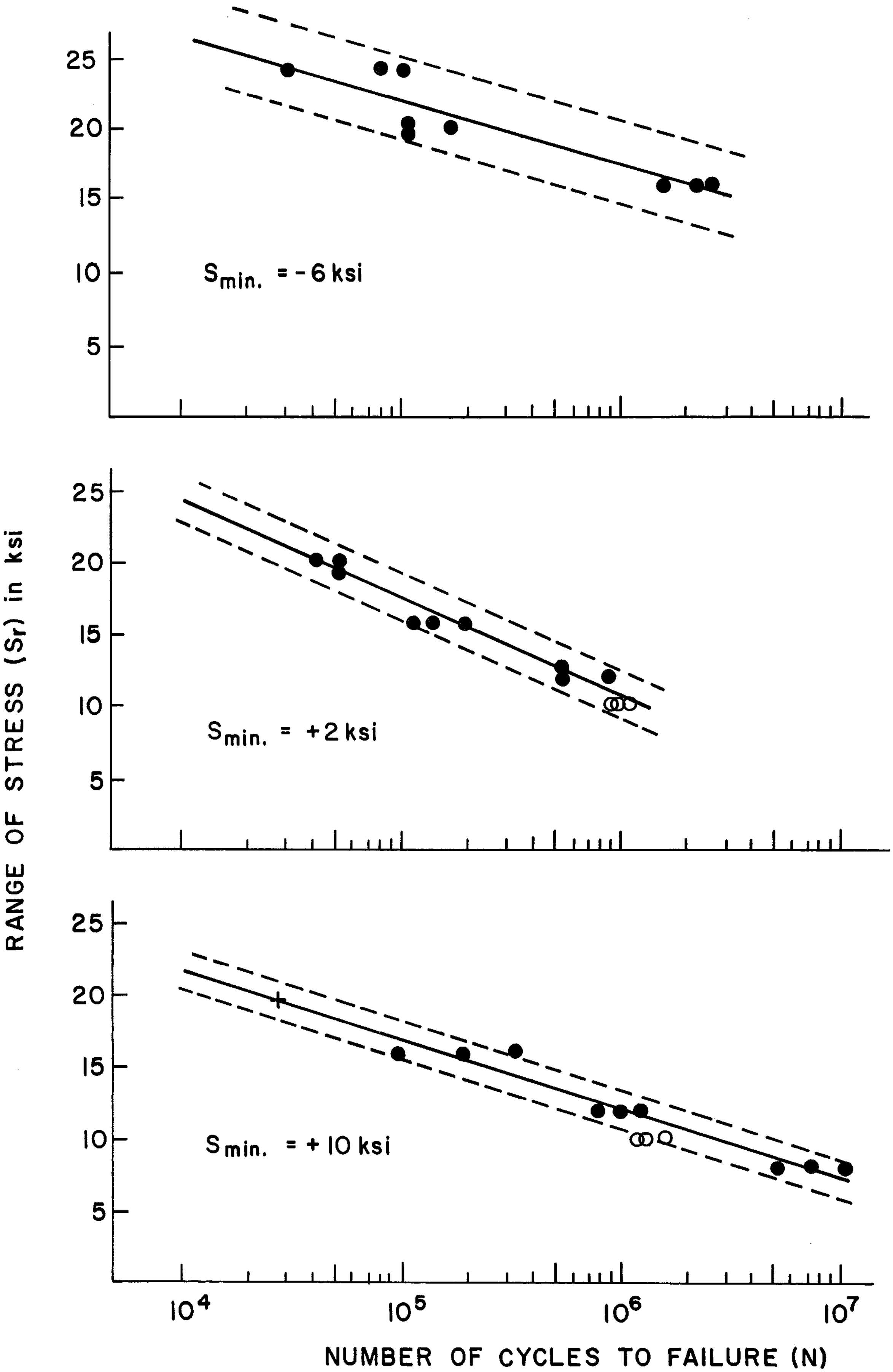


Fig. 6 S-N CURVES FOR SPECIMENS WITH 3/4-INCH DIAMETER STUDS

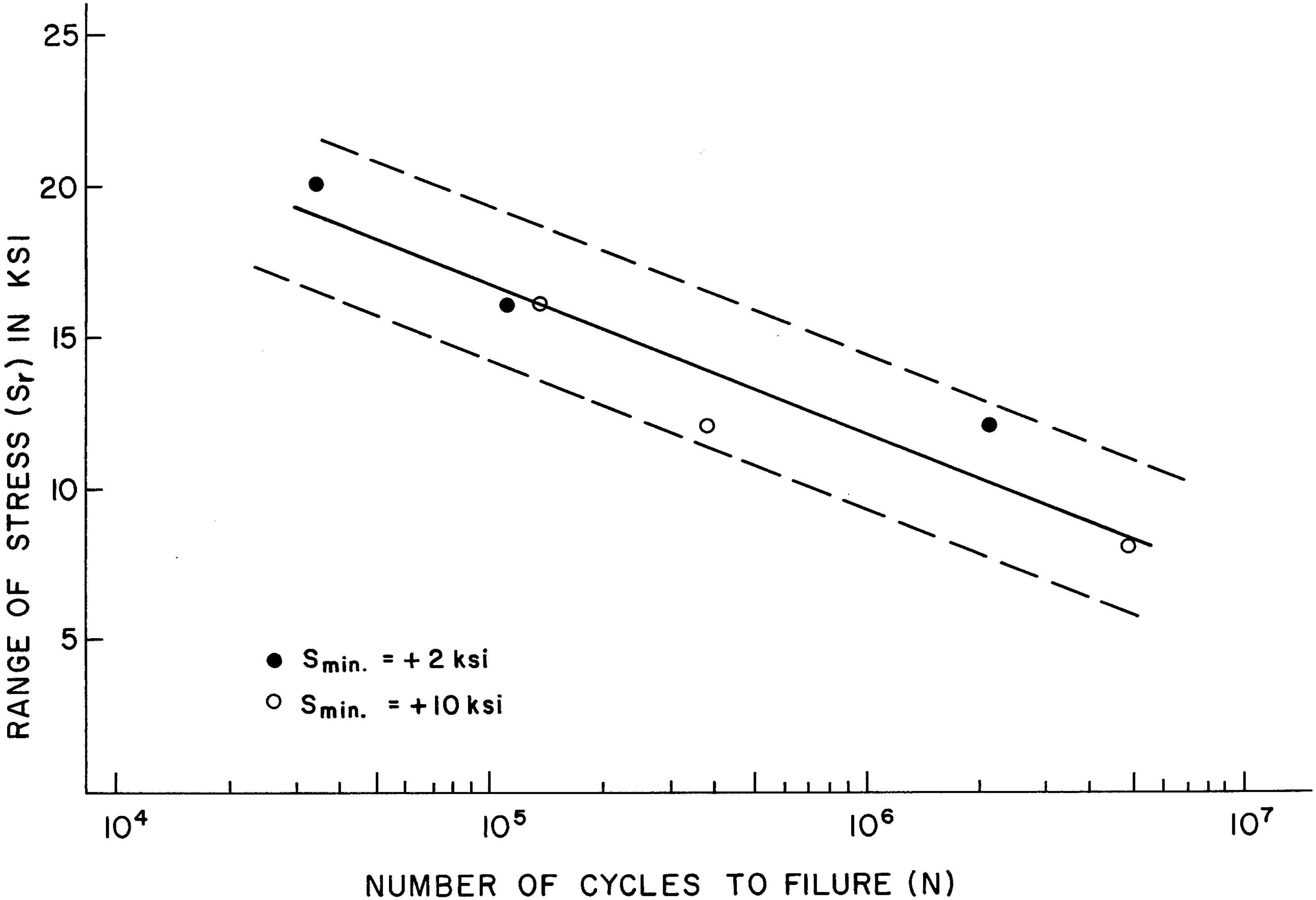


Fig. 7 S-N CURVE FOR SPECIMENS WITH 7/8-INCH DIAMETER STUDS

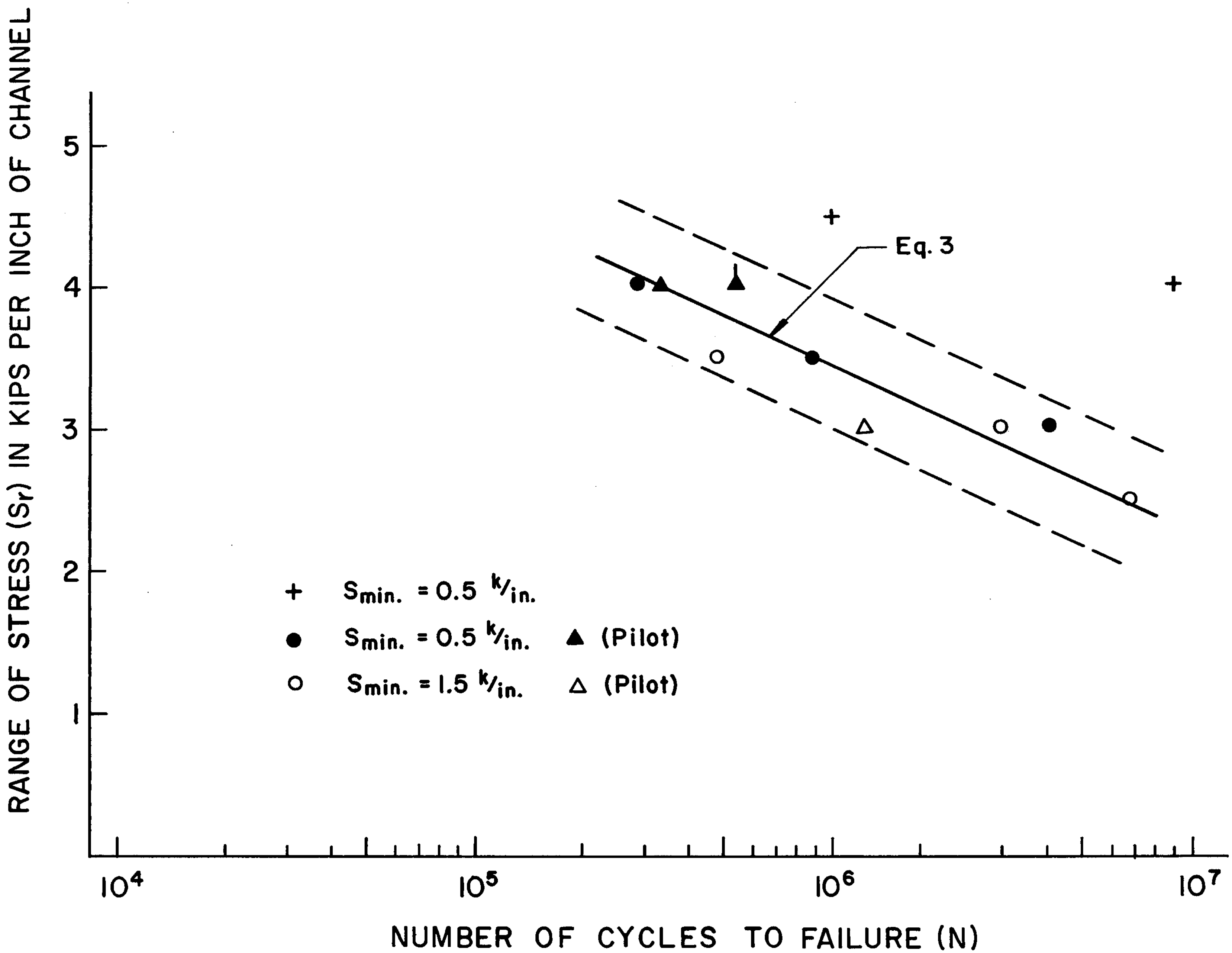


Fig. 8 S-N CURVE FOR SPECIMENS WITH 4 5.4 LB. CHANNELS

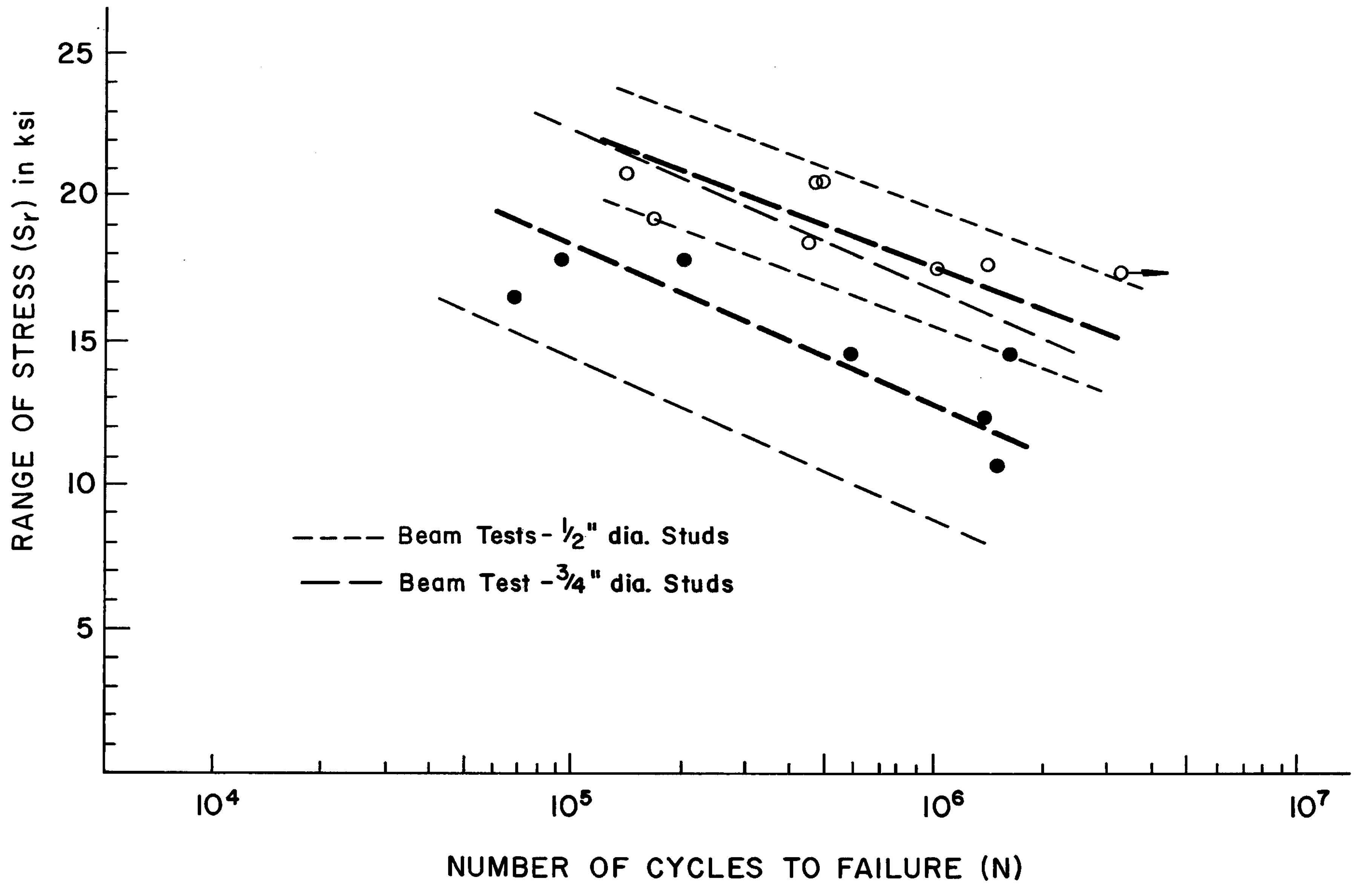


Fig. 9 S-N CURVES FOR BEAM SPECIMENS WITH 1/2-INCH AND 3/4-INCH DIAMETER CONNECTORS

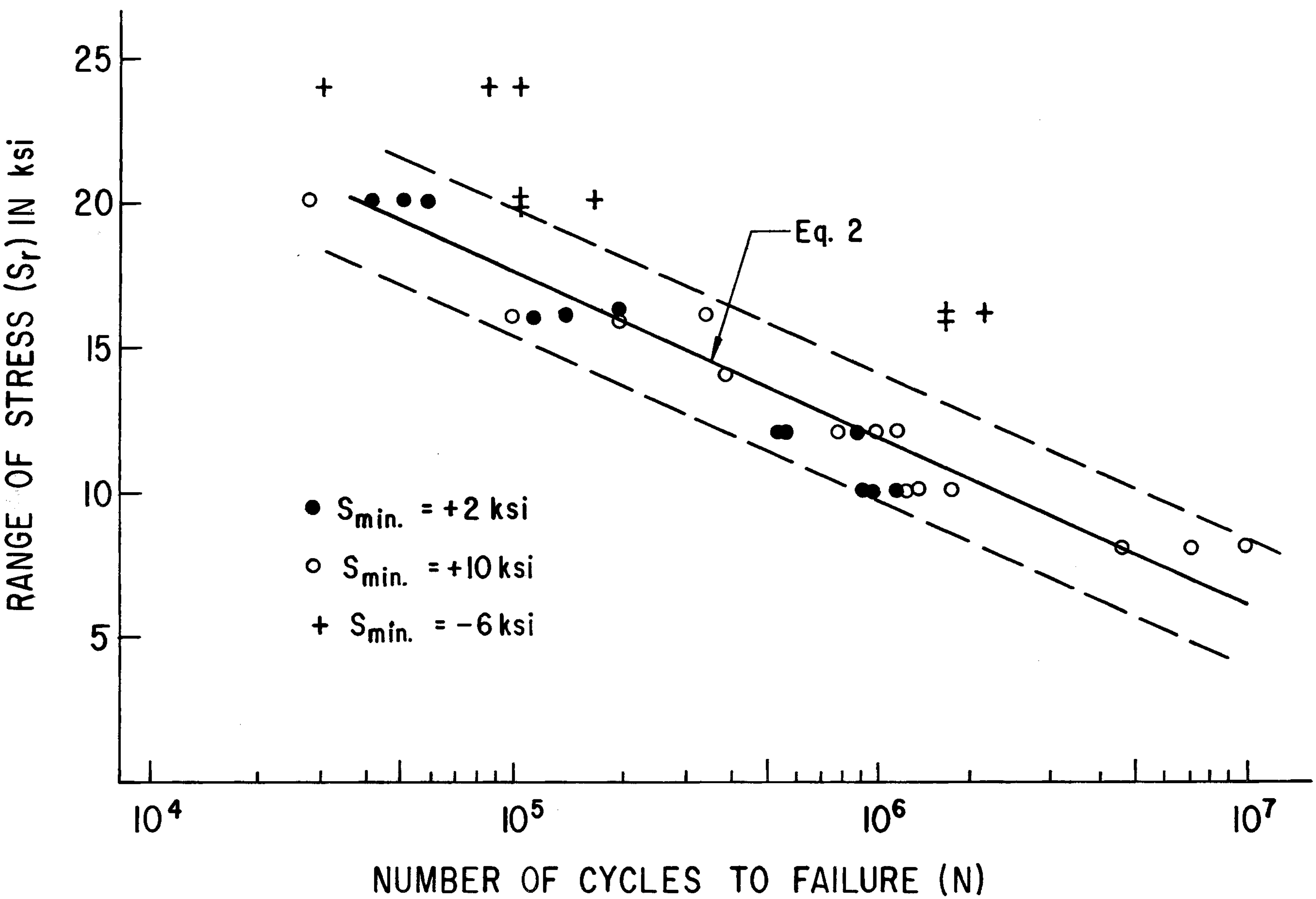


Fig. 10 REGRESSION ANALYSIS CURVE FOR 3/4-INCH DIAMETER STUDS

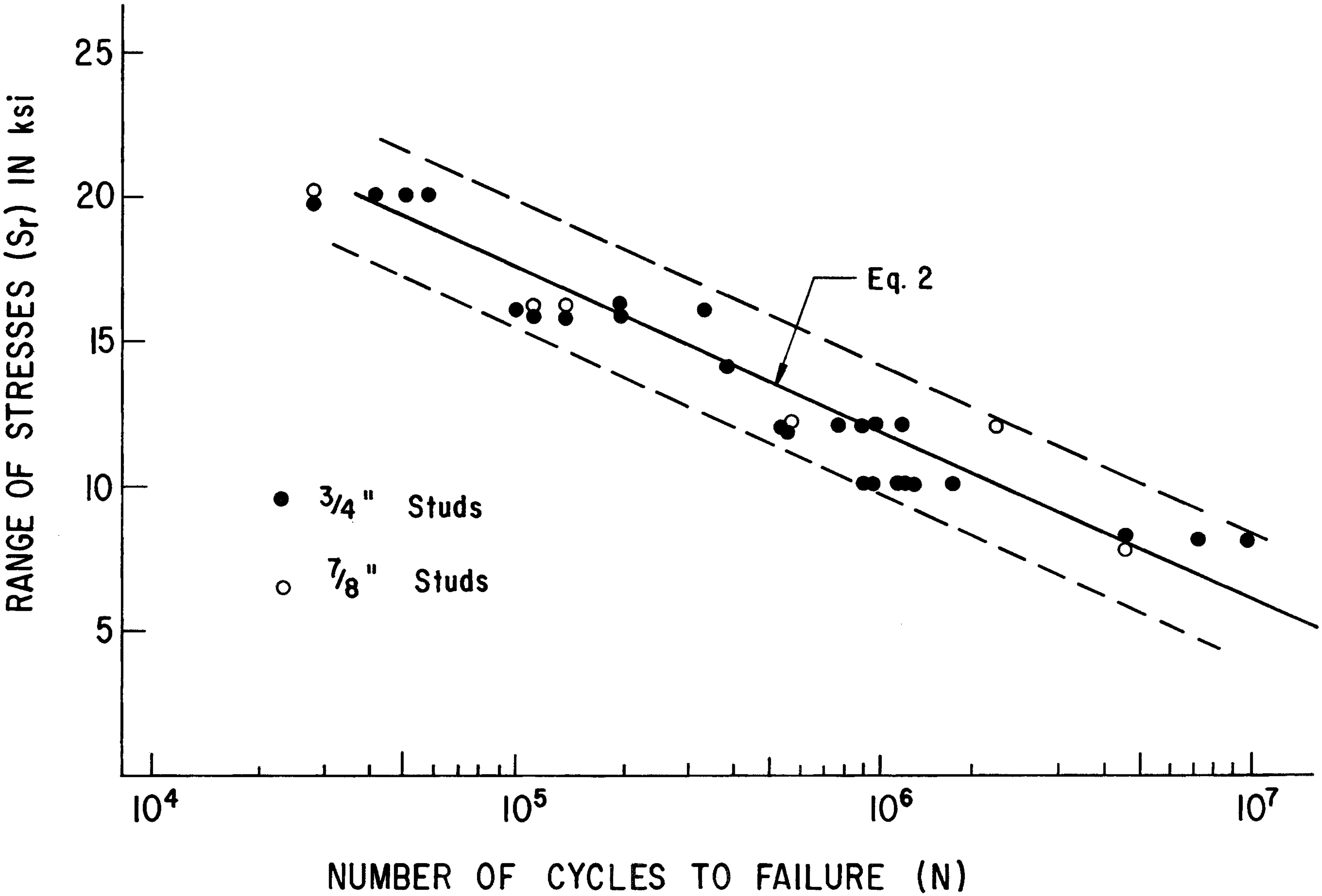


Fig. 11 COMPARISON OF PUSHOUT TEST DATA FOR 3/4-INCH AND 7/8-INCH DIAMETER STUDS

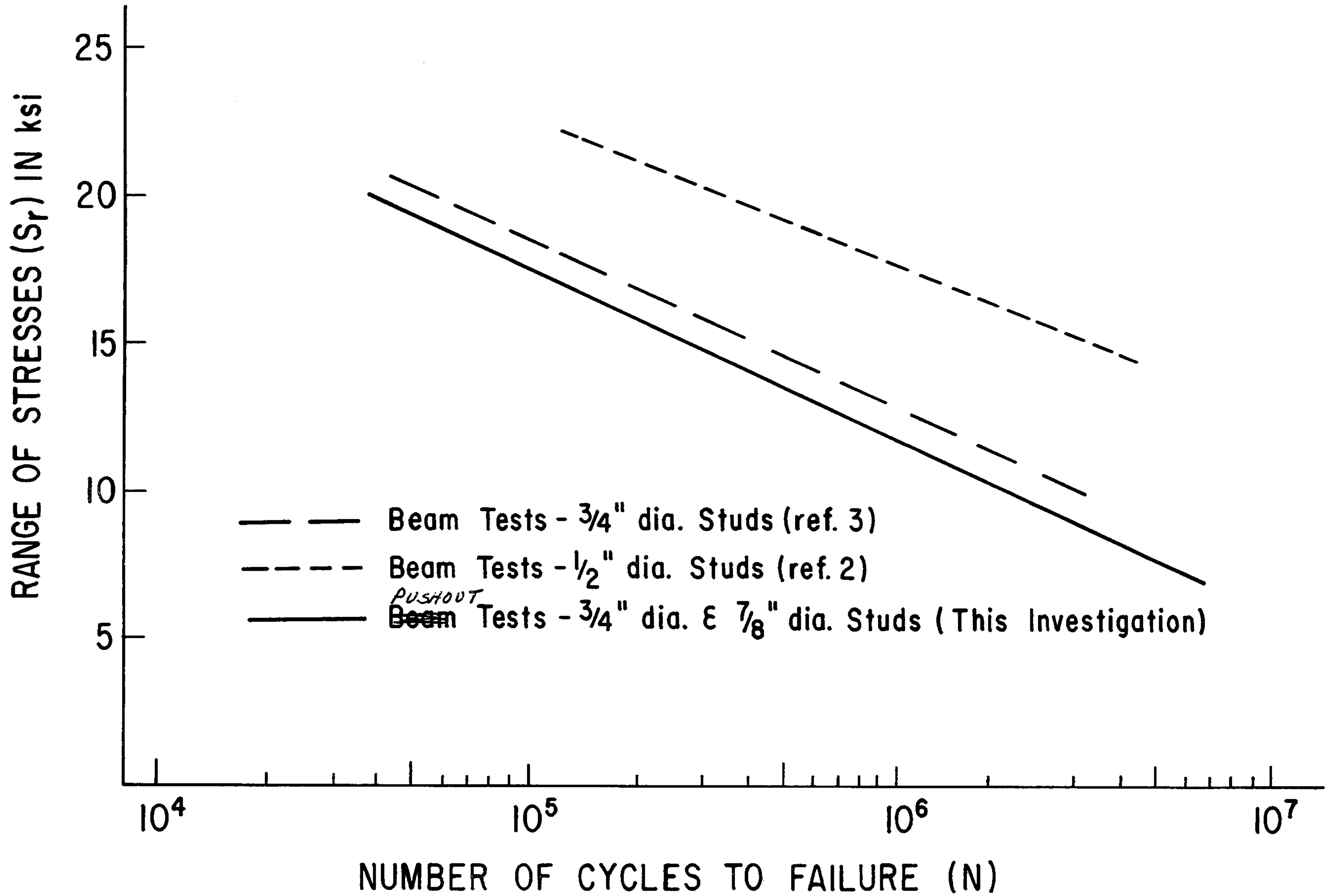


Fig. 12 COMPARISON OF REGRESSION ANALYSIS CURVES OF BEAM TESTS AND PUSHOUT TESTS

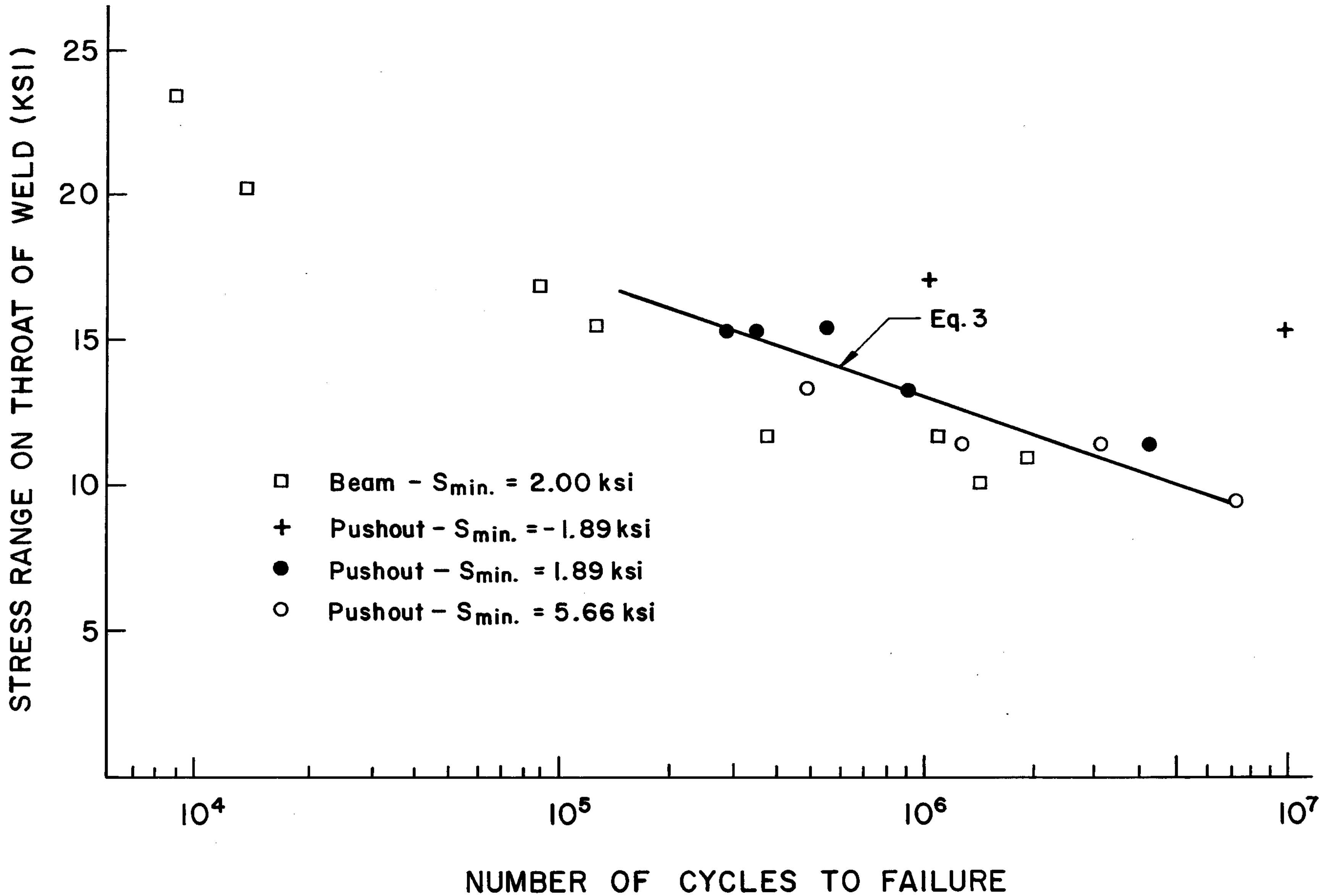


Fig. 13 COMPARISON OF BEAM AND PUSHOUT TEST DATA FOR CHANNELS

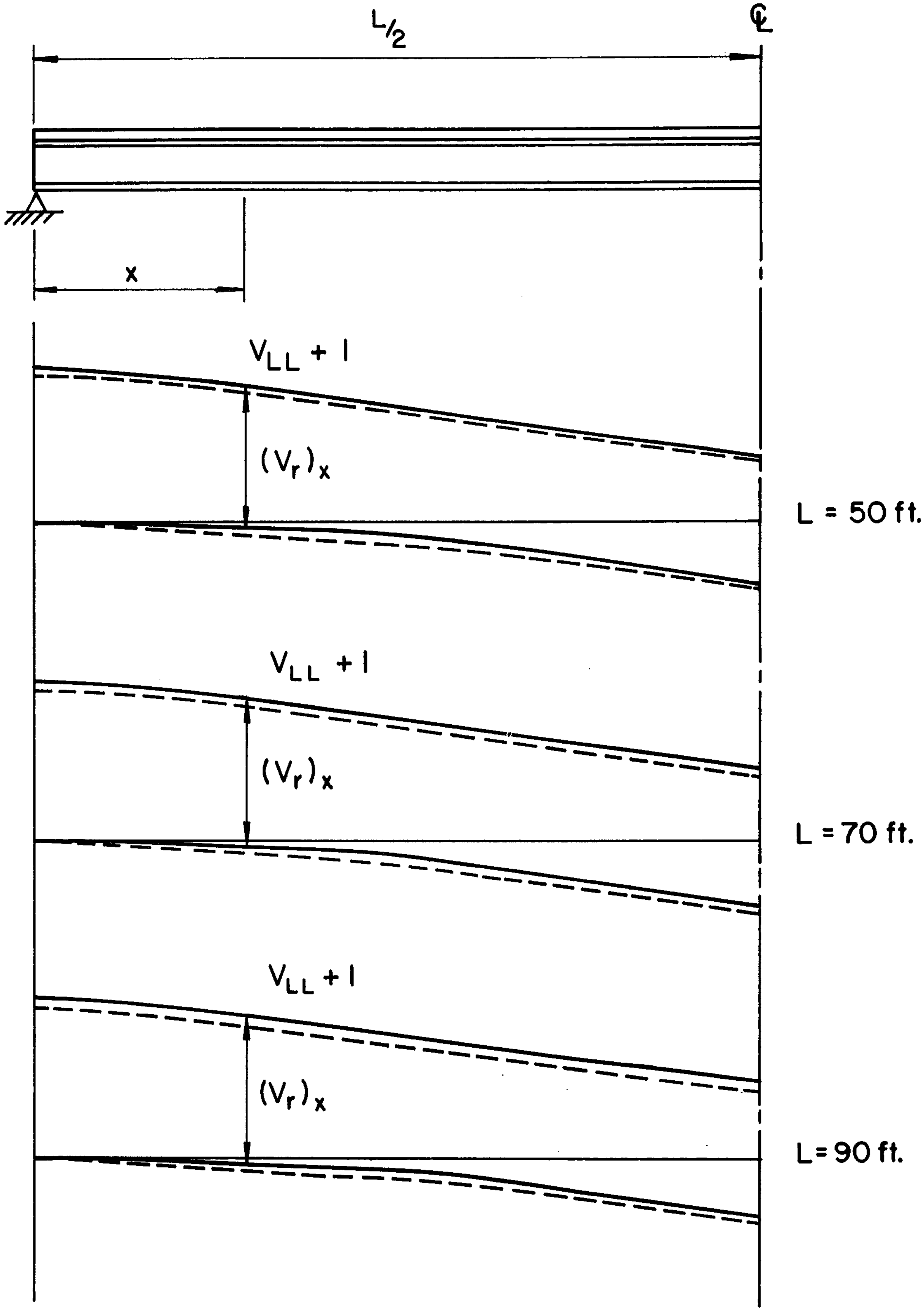


Fig. 14 SHEAR ENVELOPES FOR SIMPLE SPAN BEAMS

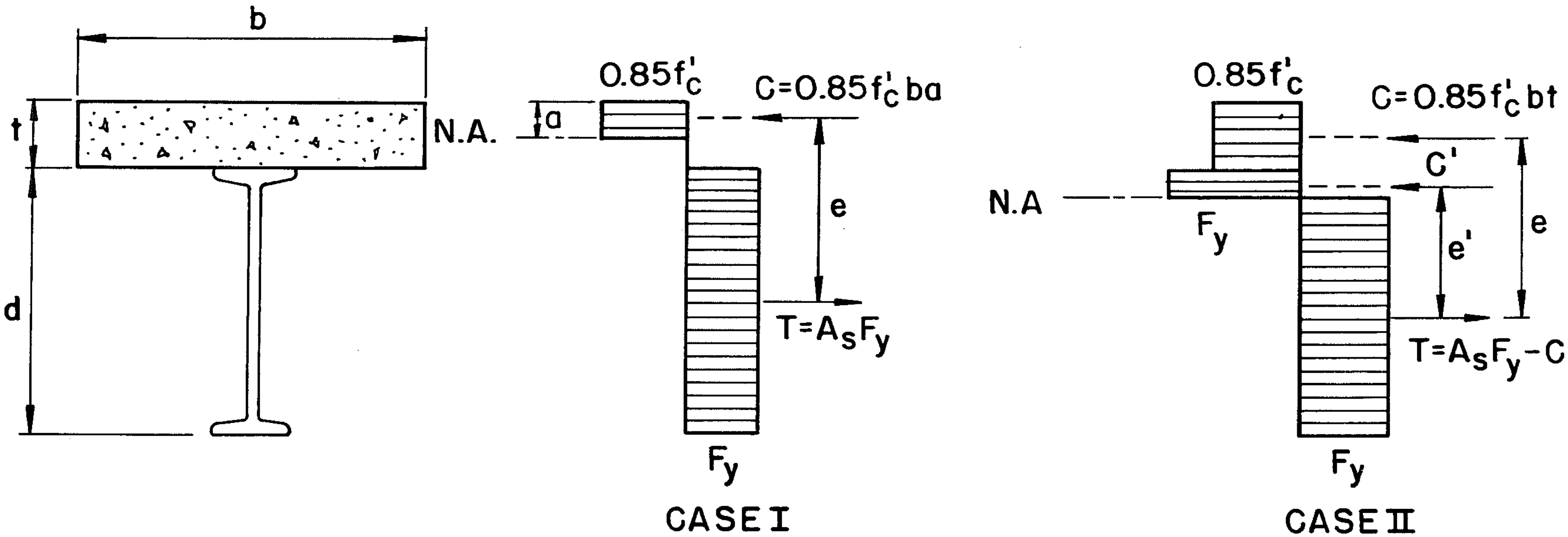


Fig. 15 STRESS DISTRIBUTION AT ULTIMATE LOAD

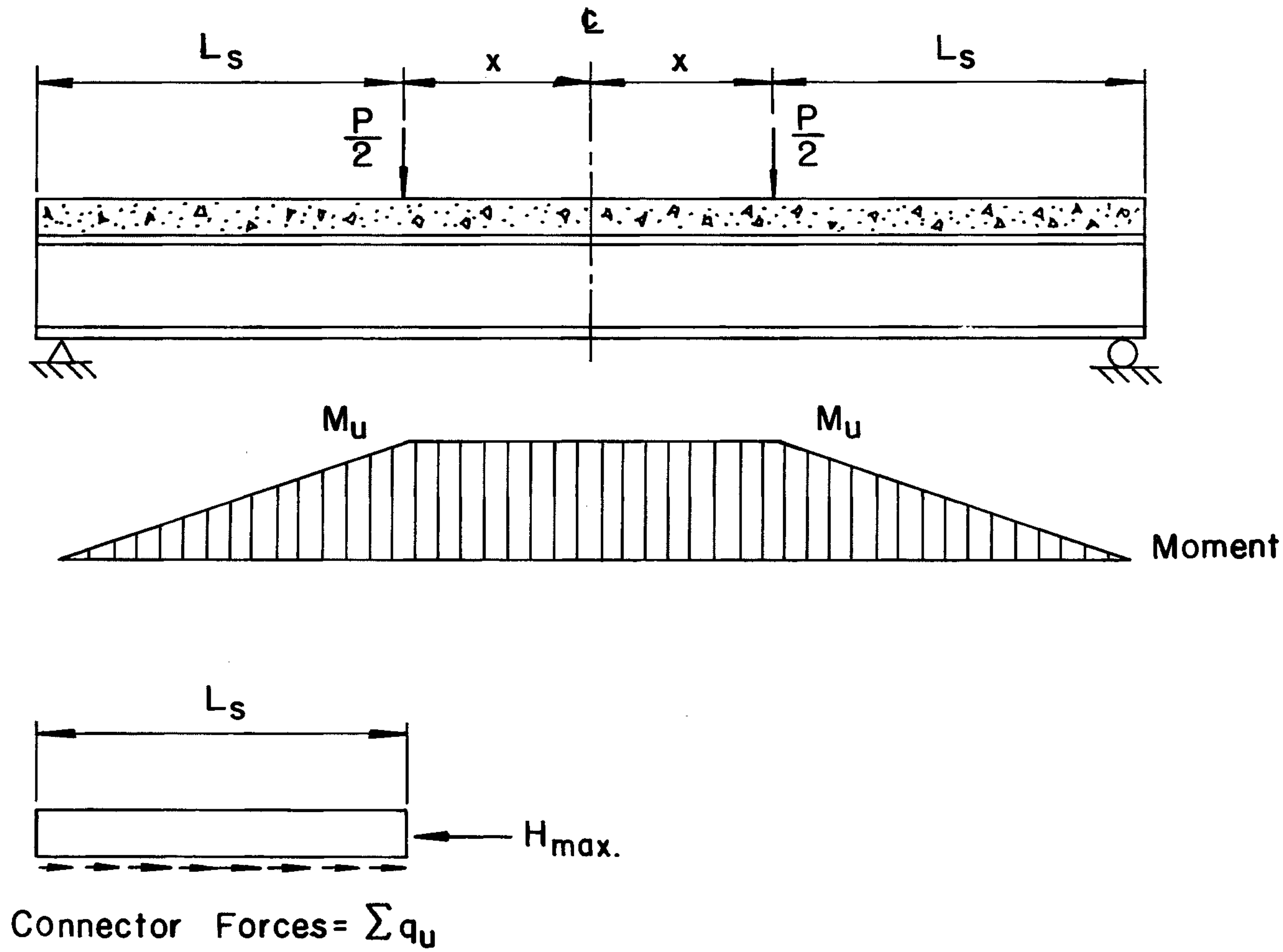


Fig. 16 SHEAR CONNECTOR FORCES AT ULTIMATE LOAD FOR A SIMPLE SPAN BEAM

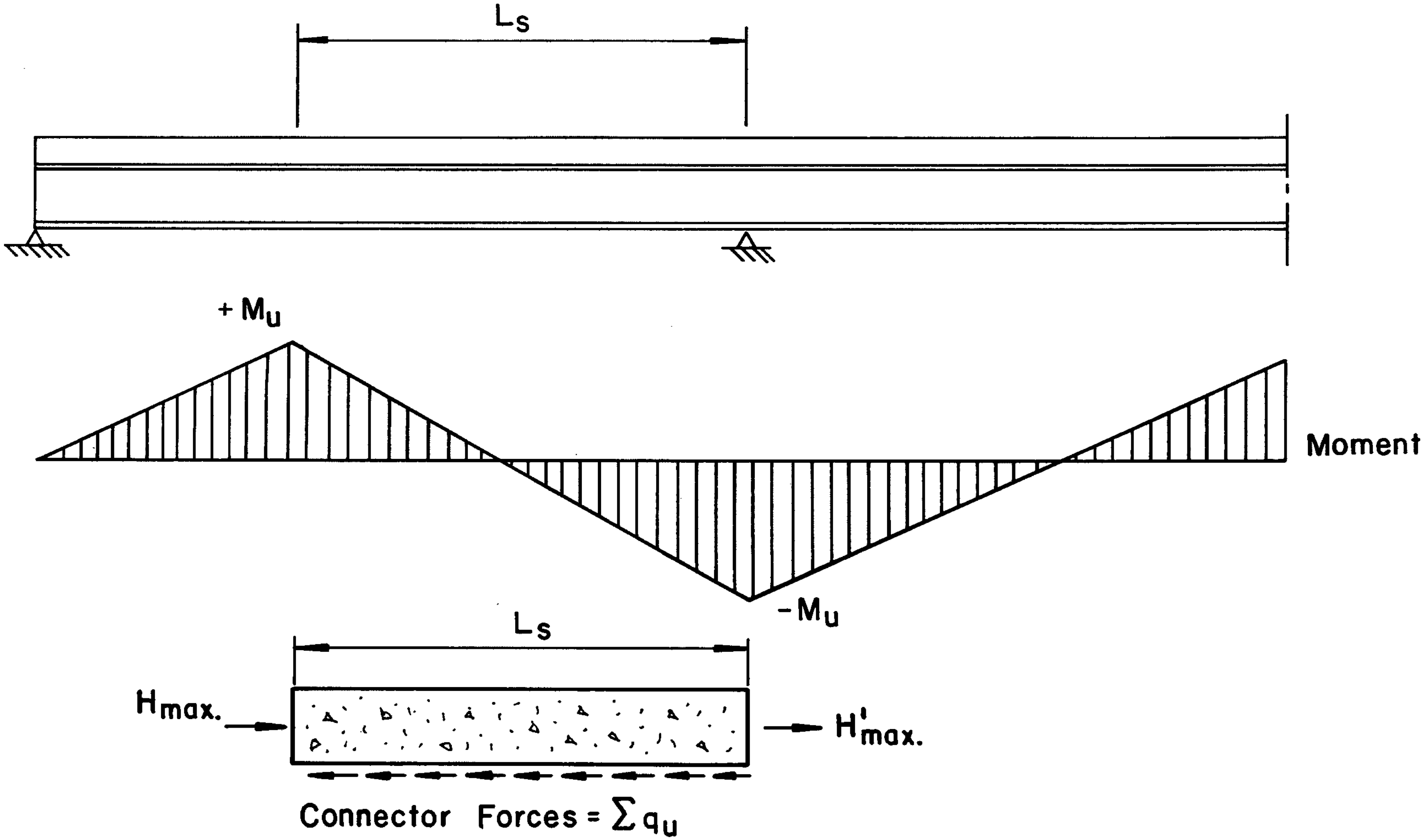


Fig. 17 SHEAR CONNECTOR FORCES AT ULTIMATE LOAD FOR A CONTINUOUS BEAM

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